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August 22, 2019

Mr. Bob Power
Seacon, LLC
165 NE Juniper Street
Issaquah, Washington 98027

Geotechnical Engineering Evaluation
MV Transportation Facilities Expansion
Union Hill Lot 3 – 188th Avenue NE
Redmond, Washington
NGA Project No. 9696C19

Dear Mr. Power:

We are pleased to submit the attached report titled **"Geotechnical Engineering Evaluation – MV Transportation Facilities Expansion – Union Hill Lot 3 - 188th Avenue NE - Redmond, Washington."** This report summarizes the existing surface and subsurface conditions within the site and provides recommendations for the proposed site development. Our services were completed in general accordance with the proposal signed by you on June 27, 2019.

The site is located along the western side of 188th Avenue NE immediately east of the properties located at 7555 NE 76th Street and 18690 NE 73rd Street. The parcel number for the property is 072506-9141. The site is a roughly rectangular-shaped parcel covering approximately 4.13 acres. The site is currently undeveloped. We understand that the proposed development plans include construction of a parking lot for shuttles and busses throughout the site with associated utility improvements. Vertical relief from adjacent roadways is to be supported mainly by grading, although a keystone block retaining wall less than 4 feet in height may be necessary within the northwestern portion of the site. The proposed finished floor elevation of the parking lot is approximately 90 ± 5 feet. The lowest portion of the site along the eastern property boundary is approximately 85 feet. Current grading plans provided for the site show excavations to bring the site to the proposed elevations. Based on groundwater elevation data and previous experience with projects in the vicinity of the site, we do not anticipate the need for dewatering of the site during construction. Specific stormwater plans were also not available at the time this report was prepared. However, we anticipate that due to the relatively silty nature and thickness of the fill soils that underlie the surface of the site that infiltration is likely not feasible and that stormwater will likely be directed to an appropriate stormwater collection system within the site.

We explored the subsurface soil and groundwater conditions on August 1, 2019 with seven trackhoe-excavated test pits. In general, the test pits exposed silty sand with gravel with varying amounts of debris to the depths explored. We interpreted the soils to be undocumented fill soils that were placed here as a part of previous grading and filling performed within the property. Review of a previously prepared geotechnical report for nearby properties indicated that the property to the east was explored with nine drilled borings extending to depths in the range of 26.5 to 46.5 feet below the existing ground surface.

These borings generally encountered undocumented fill soils consisting of lean clay, clayey sand, sandy silt, silty sand, and silty gravel with varying amounts of cobbles, boulders, organics, and wood debris within the upper portion of the borings. Seven of the nine borings were completed within the fill soils. Within the two northern borings, sands and gravels interpreted to be native recessional outwash were encountered at approximately 40 feet below the existing ground surface or an elevation of 45 feet.

We have concluded that the site is generally compatible with the planned parking lot development from a geotechnical standpoint. We understand that the proposed parking lot will likely be supported by 3H:1V graded slopes and short, keystone block retaining walls less than 4 feet in height. We have provided recommendations for pavement subgrade and pavement sections in the following report, as well as designs for the planned Keystone walls.

It has been a pleasure to provide service to you on this project. Please contact us if you have any questions regarding this report or require further information.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Khaled M. Shawish, PE
Principal

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Geotechnical Engineering Evaluation
MV Transportation Facilities Expansion
Union Hill Lot 3 – 188th Avenue NE
Redmond, Washington

INTRODUCTION

This report presents the results of our geotechnical engineering investigation and evaluation of the MV Transportation Facilities Expansion project in Redmond, Washington. The project site is known as Lot 3 Union Hill, and is located southwest of the intersection of NE 76th Street and 188th Avenue NE as shown on the Vicinity Map in Figure 1. The purpose of this study is to explore and characterize the site's surface and subsurface conditions and to provide geotechnical recommendations for the proposed site development.

We understand that the proposed development plans include construction of a parking lot for shuttles and busses throughout the site with associated utility improvements. Vertical relief from adjacent roadways is to be supported mainly by grading, although a keystone block retaining wall less than 4 feet in height may be necessary within the northwestern portion of the site. The proposed finished elevation of the parking lot is approximately 90 ± 5 feet. The lowest portion of the site along the eastern property boundary is approximately 85 feet. Current grading plans provided for the site show excavations to bring the site to the proposed elevations. Based on groundwater elevation data and previous experience with projects in the vicinity of the site, we do not anticipate the need for dewatering of the site during construction. Specific stormwater plans were also not available at the time this report was prepared. However, we anticipate that due to the relatively silty nature and thickness of the fill soils that underlie the surface of the site that infiltration is not feasible and that stormwater will likely be directed to an appropriate stormwater collection system within the site. The existing and proposed site conditions are shown on the Site Plan in Figure 2.

SCOPE

The purpose of this study is to explore and characterize the site surface and subsurface conditions, and provide general recommendations for site development. Specifically, our scope of services included the following:

1. A review of available soil and geologic maps of the area, available plans, and any available geotechnical reports for the property.
2. Exploring the subsurface soil and groundwater conditions within the site and vicinity of the proposed retaining wall alignments with 15- to 20-foot deep excavated test pits. Excavation services were provided by the Client.
3. Performing laboratory classification and analyses on soil samples obtained from the explorations, as necessary.
4. Providing recommendations for site grading and earthwork, including structural fill materials and construction standards.
5. Providing recommendations for temporary and permanent slopes.

6. Providing recommendations for pavement subgrade.
7. Providing recommendations for site drainage and erosion control.
8. Providing recommendations for retaining wall design and construction.
9. Providing calculations and engineering details for planned fill and retaining walls.
10. Documenting the results of our conclusions and recommendations in a written geotechnical engineering report.

SITE CONDITIONS

Surface Conditions

The site is located along the western side of 188th Avenue NE immediately east of the properties located at 7555 NE 76th Street and 18690 NE 73rd Street. The parcel number for the property is 072506-9141. The site is a roughly rectangular-shaped parcel covering approximately 4.13 acres. The site is currently undeveloped. Large soil stockpiles from previous grading activities are located within the southern central portion of the property.

Approximately 2 Horizontal to 1 Vertical (2H:1V) slopes are also located on the southern and northern sides of the site, supporting roughly 6 to 11 feet of vertical relief from adjacent roadway areas to the upland portions of the site, respectively. On the eastern portion of the site, 3H:1V slopes support relief of up to 7 feet from 188th Avenue NE. Based on our experience with the neighboring sites to the west, we understand that the soil stockpiles and graded slopes were created during past grading and filling activities within the site. Elevations within the site range from 73 feet within the lower northern portion of the site to 111 feet at the top of the soil stockpile within the northwestern portion of the property. An approximate elevation contour of 94 feet is located along the toe of the soil stockpiles and the top of most of the graded slopes within the northern, eastern, and southern perimeter of the property. Shallow ponding surface water was observed in the northeastern, upland portion of the site during our site visit on August 1, 2019. The water is associated with a sediment settlement pond. We did not observe signs of recent soil movement or groundwater seepage on the site fill slopes.

Subsurface Conditions

Geology: The geologic units for this area are shown on the Geologic Map of the Redmond Quadrangle, King County, Washington, by James P. Minard and Derek B. Booth (US Geological Survey, 1988). The site is mapped as Redmond Delta (Qvrd). These deposits are described as sand with gravel soils. In general, our explorations along with the previous exploration performed within the site encountered silty sand with gravel that we interpreted as previously placed structural fill during previous grading activities.

Explorations: The subsurface conditions within the site were explored on August 1, 2019 by excavating seven test pit explorations throughout the property that extended to depths of approximately 11.5 to 15.0 feet below the existing ground surface. The approximate locations of our explorations are shown on the Schematic Site Plan in Figure 2. A geologist from NGA was present during the explorations, examined the soils and geologic conditions encountered, obtained samples of the different soil types, and maintained logs of the test pits.

The soils were visually classified in general accordance with the Unified Soil Classification System, presented in Figure 3. The test pit logs are attached to this report and are presented as Figures 4 and 5. We present a brief summary of the subsurface conditions in the following paragraph. For a detailed description of the subsurface conditions, the test pit logs should be reviewed.

Test pits generally encountered a surficial layer brown, silty, fine to medium sand with gravel, organics, and varying amounts of anthropogenic debris, including brick, plastic, and processed wood. This material extended to depths of 5.5 to 8 feet below the existing surface and was encountered in a loose to medium dense condition. We interpreted this deposit to be recent undocumented fill. Underlying the recent fill, explorations recovered silt with fine sand and gravel, cobbles, boulders, and organic debris in a loose to medium dense condition. We interpreted this material to be undocumented fill associated with historic site grading after completion of surface mining operations. In Test Pit 5 on the northeastern portion of the site, we encountered dark brown to black silty, fine to coarse sand with gravel and anthropogenic debris to a depth of 15 feet below the surface, where the exploration was terminated. We interpreted this soil to be fill of an abandoned temporary sediment control pond on the site.

Deeper subsurface boring explorations were performed within adjacent property to the northwestern portion of the site by Kleinfelder in 2015. This exploration program consisted of nine drilled borings extending down to depths of 26.5 to 46.5 feet below the existing ground surface. These borings generally encountered undocumented fill soils consisting of lean clay, clayey sand, sandy silt, silty sand, and silty gravel with varying amounts of cobbles, boulders, organics, and wood debris within the upper portion of the borings. Seven of the nine borings were completed within the fill soils. Within the two northern borings, sands and gravels interpreted to be native recessional outwash were encountered at approximately 40 feet below the existing ground surface or an elevation of 45 feet. These two borings were terminated within the native recessional outwash soils.

Hydrogeologic Conditions

Groundwater seepage was encountered in the explorations where historic sediment settlement ponds had been present, specifically in Test Pits 5 and 6 at depths of 10 and 5.5 feet below the surface, respectively. We did not observe groundwater emitting from the site slopes. The groundwater table on this site is interpreted to be well below any proposed modifications to the site. Any near-surface groundwater encountered on this site, such as that which was encountered in explorations would be interpreted as a perched water condition. Perched water occurs when surface water infiltrates through less dense, more permeable soils and accumulates on top of underlying, less permeable soils. Perched water does not represent a regional groundwater "table" within the upper soil horizons. Perched water tends to vary spatially and is dependent upon the amount of precipitation. We would expect the amount of perched water to decrease during drier times of the year and increase during wetter periods.

SENSITIVE AREA EVALUATION

Seismic Hazard

The 2018 International Building Code (IBC) seismic design section provides a basis for seismic design of structures. Since medium dense/medium stiff or better glacial soils were encountered underlying the site at depth, the site conditions best fit the IBC description for Site Class D. Table 1 below provides seismic design parameters for the site that are in conformance with the 2015 IBC, which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years), and the 2008 USGS seismic hazard maps.

Table 1 – 2015 IBC Seismic Design Parameters

Site Class	Spectral Acceleration at 0.2 sec. (g) S_S	Spectral Acceleration at 1.0 sec. (g) S_1	Site Coefficients		Design Spectral Response Parameters (g)	
			F_a	F_v	S_{DS}	S_{D1}
D	1.249	0.478	1.001	1.522	0.833	0.485

The spectral response accelerations were obtained from the USGS Earthquake Hazards Program Interpolated Probabilistic Ground Motion website (2008 data) for the project latitude and longitude.

Hazards associated with seismic activity include liquefaction potential and amplification of ground motion by soft deposits. Liquefaction is caused by a rise in pore pressures in a loose, fine sand deposit beneath the groundwater table. The particle size distribution of materials in the undocumented fills on the site result in a low potential for liquefaction at the site. The proposed parking lot should not experience detrimental effects of amplification of ground motion if recommendations for subgrade improvements are followed as specified in this report.

The loose surficial materials and undocumented fill soils on the site slopes currently have the potential for shallow sloughing failures during seismic events. Such events should not affect the proposed parking lot, provided the site is graded and designed in accordance with the recommendations presented in this report.

Erosion Hazard

The criteria used for determination of the erosion hazard for affected areas include soil type, slope gradient, vegetation cover, and groundwater conditions. The erosion sensitivity is related to vegetative cover and the specific surface soil types, which are related to the underlying geologic soil units. The Soil Survey of King County Area, Washington, by the Soil Conservation Service (SCS), was reviewed to determine the erosion hazard of the on-site soils. The surface soils for this site are mapped as Everett very gravelly sandy loam, 8 to 15 percent slopes. The erosion hazards for these soil types is listed as slight. We anticipate that the existing fill soils within the site have a moderate erosion hazard. A soil management plan for erosion at the site has been submitted under a separate cover. We have included general recommendations for erosion control in the **Erosion Control** subsection of this report.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion that the site is compatible with the planned parking lot development from a geotechnical standpoint. Our explorations and review of the site conditions indicated that the site is underlain by as much as 50 feet of previously placed fill soils. This fill is generally in a loose to medium dense condition. We understand that the proposed development will consist of a parking lot with associated utility improvements. It is also our understanding that up to 4-foot tall reinforced-earth retaining walls will be constructed along the northern and northwestern portions of the property to bring the site up to the proposed finished grade elevations. In our opinion, a geo-grid reinforced Keystone block retaining wall is suitable for the site conditions along northwestern portion of the site to support the upper parking lot area.

Our explorations and review of the previous explorations within the site generally indicated that the planned 4-foot tall wall areas and site slopes are generally underlain by previously placed fill soils. Medium dense or better fill soils should provide adequate support for the planned retaining walls. Wall foundations should be overexcavated by a minimum of one foot and backfilled with 2- to 4-inch rock spalls. We recommend that level benches be graded into the site slopes to allow for placement of the wall components and fill to be retained by the walls. NGA should be retained to review project plans prior to construction and should be retained to observe wall construction to verify wall installation is performed in accordance with the plans and our recommendations.

Subgrade preparation in the pavement areas should consist of over-excavating by a minimum of one foot, placement of a reinforcing geogrid, and replacement with crushed rock. The crushed rock should consist of a minimum 12 inches of clean 1¼-inch angular crushed rock and be compacted to structural fill specifications prior to placing pavement. We recommend that the exposed subgrade be compacted to a non-yielding condition using a heavy vibratory drum roller prior to placing the geogrid and crushed rock. The resulting surface should be proof-rolled using a loaded dump truck. Areas observed to pump or weave during the proof-roll test should be over-excavated and replaced with rock spalls. Once a stable subgrade is achieved, the geogrid and crushed rock fill could be placed over the prepared subgrade.

Underground utilities should be planned and implemented as to not interfere with geogrid placement. All utilities should be in place prior to placing geogrid. Once placed, the geogrid should never be cut or disturbed in any way. Underground utilities should be supported on a minimum of one foot of pit run. Some of the on-site soils may be suitable for use as utility trench backfill but that will be highly dependent on material makeup. This can be determined during construction under the supervision of NGA.

The soils that are expected to be encountered during site development are considered highly moisture-sensitive and will disturb in wet conditions. We recommend that the site be developed during the dry season. If construction takes place during the rainy months, the site soils may disturb and become extremely difficult to work. Also, if construction takes place during the wet season, additional expenses and delays should be expected. Additional expenses could include the need for placing a blanket of rock spalls on exposed subgrades, construction traffic areas, and pavement areas prior to placing structural fill. NGA should be retained to determine if some of the on-site soils could be used as structural fill material during construction.

All grading operations and drainage improvements planned as part of this project should be planned and completed in a manner that enhances the stability of the site, not reduces it. Any excavation spoils generated during site improvements should not be stockpiled on site but rather promptly hauled away. Also, all current and future runoff generated within the site should be collected and routed to a permanent discharge location at the bottom of the slope, or to an approved drainage system. Under no circumstances should water be allowed to concentrate or flow uncontrollably over the walls or slope. The vegetation cover on the slope should be evaluated for compatibility with desired slope stability conditions, and a vegetation management plan should be devised to enhance slope stability.

Erosion Control and Slope Protection

The on-site soils have a moderate potential for water erosion when exposed, but the actual erosion potential will be dependent on how the site is graded and how water is allowed to concentrate. Best Management Practices (BMPs) should be used to control erosion. Areas disturbed during construction should be protected from erosion. Erosion control measures may include diverting surface water away from the stripped areas. Silt fences or straw bales could be erected to prevent muddy water from flowing off the site. Stockpiles should be covered with plastic sheeting. Disturbed areas should be planted as soon as practical and the vegetation should be maintained until it is established. The erosion potential for areas not stripped of vegetation should be low. Final grading should incorporate permanent erosion control measures and should be designed to route stormwater runoff to appropriate discharge locations away from the structures and sloping ground.

Temporary and Permanent Slopes

Cuts associated with over-excavation of utility areas may be used for this project. Temporary cut slope stability is a function of many factors, including the type and consistency of soils, depth of the cut, surcharge loads adjacent to the excavation, length of time a cut remains open and the presence of surface or groundwater. It is exceedingly difficult under these variable conditions to estimate a stable, temporary, cut slope angle. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations since they are continuously at the job site, able to observe the nature and condition of the cut slopes, and able to monitor the subsurface materials and groundwater conditions encountered.

The following information is provided solely for the benefit of the owner and other design consultants and should not be construed to imply that Nelson Geotechnical Associates, Inc. assumes responsibility for job site safety. Job site safety is the sole responsibility of the project contractor.

For planning purposes, we recommend that temporary cuts in the on-site soils be no steeper than 2 Horizontal to 1 Vertical (2H:1V) if worker access is necessary. If significant groundwater seepage is encountered, we would expect that flatter inclinations would be necessary. We recommend that cut slopes be protected from erosion. These erosion protection measures may include covering cut slopes with plastic sheeting and diverting surface runoff away from the top of cut slopes. We do not recommend vertical slopes for cuts deeper than four feet, if worker access is necessary. We recommend that cut slope heights and inclinations conform to appropriate OSHA/WISHA regulations.

Permanent cut and fill slopes should be no steeper than 3H:1V, in accordance with City of Redmond regulations. However, flatter inclinations may be required in areas where loose soils are encountered. Permanent slopes should be planted and the vegetative cover should be maintained until it is established. We should review plans and visit the site to evaluate excavations for this project.

Site Preparation and Grading

After erosion control measures are implemented, site preparation should consist of overexcavating the pavement subgrade by a minimum of 12 inches as discussed in this report, and replacing the overexcavation with geogrid-reinforced structural fill. The subgrade should be proof-rolled and repaired to achieve a non-yielding state prior to placing geogrid. Level benches should be created for retaining wall and associated backfill placement. Retaining wall foundations should be supported on a minimum of one foot of rock spalls.

If, after site stripping, the ground surface should appear to be loose, it should be compacted to a non-yielding condition and then proof-rolled with a heavy rubber-tired piece of equipment. Areas observed to pump or weave during the proof-roll test should be overexcavated and replaced with rock spalls. If significant surface water flow is encountered during construction, this flow should be diverted around areas to be developed, and the exposed subgrades should be maintained in a semi-dry condition.

If wet conditions are encountered, alternative site stripping and grading techniques might be necessary. These could include using large excavators equipped with wide tracks and a smooth bucket to complete site grading and covering exposed subgrade with a layer of crushed rock for protection. If wet conditions are encountered or construction is attempted in wet weather, the subgrade should not be compacted as this could cause further subgrade disturbance. In wet conditions it may be necessary to cover the exposed subgrade with a layer of crushed rock as soon as it is exposed to protect the moisture sensitive soils from disturbance by machine or foot traffic during construction. The prepared subgrade should be protected from construction traffic and surface water should be diverted around prepared subgrade.

The site soils are considered to be moisture-sensitive and can disturb easily when wet. We recommend that construction take place during the drier summer months if possible. However, if construction takes place during the wet season, additional expenses and delays should be expected due to the wet conditions. Additional expenses could include the need for placing a blanket of rock spalls on exposed subgrades, construction traffic areas, and paved areas prior to placing structural fill. The successful use of on-site soils as structural fill will be very difficult, but will depend on the moisture content of the soil at the time of construction. NGA should be retained to determine if any of the on-site soils could be used as structural fill material prior to construction.

Keystone Block Retaining Wall

The total height of the new block walls will vary somewhat, but we understand that they will generally be up to approximately four feet in exposed heights. We have provided a wall design for an up to 6-foot-tall total retaining wall with geogrid-reinforced fill utilizing 21.5-inch Standard Keystone blocks. The retained fill zone should consist of imported granular material compacted to structural fill specifications. The drainage system, as indicated on the detail, should be installed along the base of the blocks and behind the wall facing.

Traffic surcharge loads of 250 psf were included in the overall Keystone Block Wall design to account for heavy-traffic loading. The surcharge load was applied to the Keystone Block wall design and setback 5.0 feet back from the face of the Keystone block wall. A geogrid-reinforced wall detail and construction notes are shown in Figure 6. Please refer to Appendix A for detailed Keystone Retaining wall calculations.

The block facing should consist of 21.5-inch Standard Keystone blocks. The block facing should be placed on a minimum of 6-inch thick crushed rock leveling pad placed over a minimum of one foot of 2- to 4-inch rock spalls. The subgrade should be level and compacted to a non-yielding condition before placing the blocks or backfill.

A drainage blanket of 12 inches of free-draining ¾-inch clean crushed rock should be placed between the blocks and the retained fill zone. The block cavities should also be filled with the crushed rock. A rigid, 6-inch perforated drainpipe embedded in a minimum of one foot of drain rock and wrapped in a filter fabric should be placed at the bottom of the drainage blanket. The drain should be sloped to drain into an approved system.

Mirafi 5XT geogrid (or equivalent) is recommended to be used in the geogrid-reinforced fill wall design. The geogrid should be cut to the recommended lengths, attached to the blocks as recommended by the manufacturer, and extended back into the reinforced fill zone. The grid should be pulled tight before the fill is placed over the geogrid. Care should be taken to not damage the geogrid by operating construction equipment on the exposed grid, or by allowing large rocks to be placed directly on the grid.

All fill placed in the retained fill zone behind the retaining walls should be placed in accordance with the recommendations laid out in the **Structural Fill** subsection of this report.

If groundwater seepage is encountered or if excessive rainfall occurs during construction of specific aspects, we recommend that the contractor slope the bottom of the excavations and direct the water to ditches and small sump pits. The collected water can then be directed to a suitable discharge point.

Structural Fill

General: Fill placed behind retaining walls and underneath pavement areas should be placed as structural fill. Structural fill, by definition, is placed in accordance with prescribed methods and standards, and is monitored by an experienced geotechnical professional or soils technician. Field monitoring procedures would include the performance of a representative number of in-place density tests to document the attainment of the desired degree of relative compaction. The area to receive the fill should be suitably prepared as described in the **Site Preparation and Grading** subsection of this report, prior to beginning fill placement. Sloping areas on this site should be benched for fill placement. The benches should be level and be a minimum of six feet in width.

Materials: Structural fill should consist of a good quality, all-weather granular soil, free of organics and other deleterious material and be well graded to a maximum size of about three inches. Fill material should contain no more than five-percent fines (soil finer than U.S. No. 200 sieve, based on that fraction passing the U.S. 3/4-inch sieve). The use of the on-site soils as structural fill is not recommended. NGA should be retained during construction to determine if any of the on-site soils could be used as structural fill.

Fill Placement: Following subgrade preparation, placement of structural fill may proceed. All backfilling should be accomplished in uniform lifts up to eight inches thick. Each lift should be spread evenly and be thoroughly compacted prior to placement of subsequent lifts. All structural fill should be compacted to a minimum of 95 percent of its maximum dry density. Maximum dry density, in this report, refers to that density as determined by the ASTM D-1557 Compaction Test procedure. The moisture content of the soils to be compacted should be within about two percent of optimum so that a readily compactable condition exists. It may be necessary to over-excavate and remove wet soils in cases where drying to a compactable condition is not feasible. All compaction should be accomplished by equipment of a type and size sufficient to attain the desired degree of compaction.

Pavements

Pavement subgrade preparation, and structural fill placement should be completed as recommended in the **Site Preparation and Grading** and **Structural Fill** subsections of this report. We recommend that a minimum of 12-inches of clean 1¼-inch crushed rock be placed below the pavement section, underlain by a Tensar TX 160 geogrid, or equivalent. The existing soil should be over excavated and replaced with crushed rock fill prior to placing new pavement section. The pavement subgrade should be heavily compacted and proof-rolled with a heavy, rubber-tired piece of equipment, to identify soft or yielding areas that require repair prior to placing geogrid and crushed rock. We should be retained to observe the proof-rolling and recommend subgrade repairs prior to placement of the geogrid and crushed rock.

We recommend the pavement section consist of the eight inches of crushed rock base-course, overlain by 4.0 inches of PG 64-22 Class ½-inch Hot Mix Asphalt (HMA). The base-course layer is in addition to the 1¼-inch crushed rock.

Utilities

We recommend that underground utilities be underlain with a minimum 12 inches of pit run prior to backfilling the trench with on-site or imported material meeting structural fill requirements. Trenches within settlement sensitive areas should be compacted to 95% of the modified proctor as described in the **Structural Fill** subsection of this report. Trenches located in non-structural areas should be compacted to a minimum 90% of the maximum dry density. When excessively soft and/or debris-laden soils are encountered within utility trench excavations, such soils should be overexcavated and replaced with crushed rock. All underground utilities need to be in place prior to geogrid placement.

Site Drainage

Surface Drainage: The finished ground surface should be graded such that stormwater is directed to an appropriate stormwater collection system. Surface water should be collected by permanent catch basins and drain lines, and be discharged into an appropriate discharge system.

Subsurface Drainage: If perched groundwater is encountered during construction, we recommend that the contractor slope the bottom of the excavation and collect the water into ditches and small sump pits where the water can be pumped out of the excavation and routed into an appropriate discharge point.

We recommend the use of drains behind retaining walls. The drains should consist of a minimum four-inch-diameter, rigid, slotted or perforated, PVC pipe surrounded by free-draining material, such as washed rock, wrapped in a filter fabric. We recommend that an 18-inch-wide zone of clean (less than three-percent fines), granular material be placed along the back of subsurface walls above the drain. Pea gravel is an acceptable drain material, or drainage composite may be used instead. The free-draining material should extend up the wall to one foot below the finished surface. The top foot of backfill should consist of impermeable soil placed over plastic sheeting or building paper to minimize the migration of surface water or fines into the footing drain. Footing drains should discharge into tightlines leading to an appropriate collection and discharge point with convenient cleanouts to prolong the useful life of the drains.

USE OF THIS REPORT

NGA has prepared this report for Mr. Bob Power and his agents, for use in the planning and design of the development planned on this site only. The scope of our work does not include services related to construction safety precautions and our recommendations are not intended to direct the contractors' methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. There are possible variations in subsurface conditions between the explorations and also with time. Our report, conclusions, and interpretations should not be construed as a warranty of subsurface conditions. A contingency for unanticipated conditions should be included in the budget and schedule.

We recommend that NGA be retained to review project plans and consult with the design team during final design. We also recommend that NGA be retained to provide monitoring and consultation services during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications. We should be contacted a minimum of one week prior to construction activities and could attend pre-construction meetings if requested.

Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering practices in effect in this area at the time this report was prepared. No other warranty, expressed or implied, is made. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the owner.

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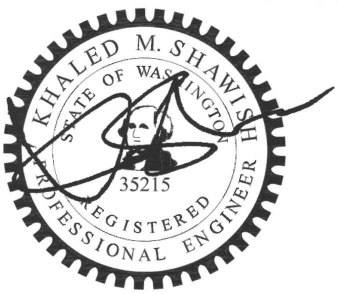
It has been a pleasure to provide service to you on this project. If you have any questions or require further information, please call.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Carston T. Curd, GIT
Project Geologist



Khaled M. Shawish, PE
Principal

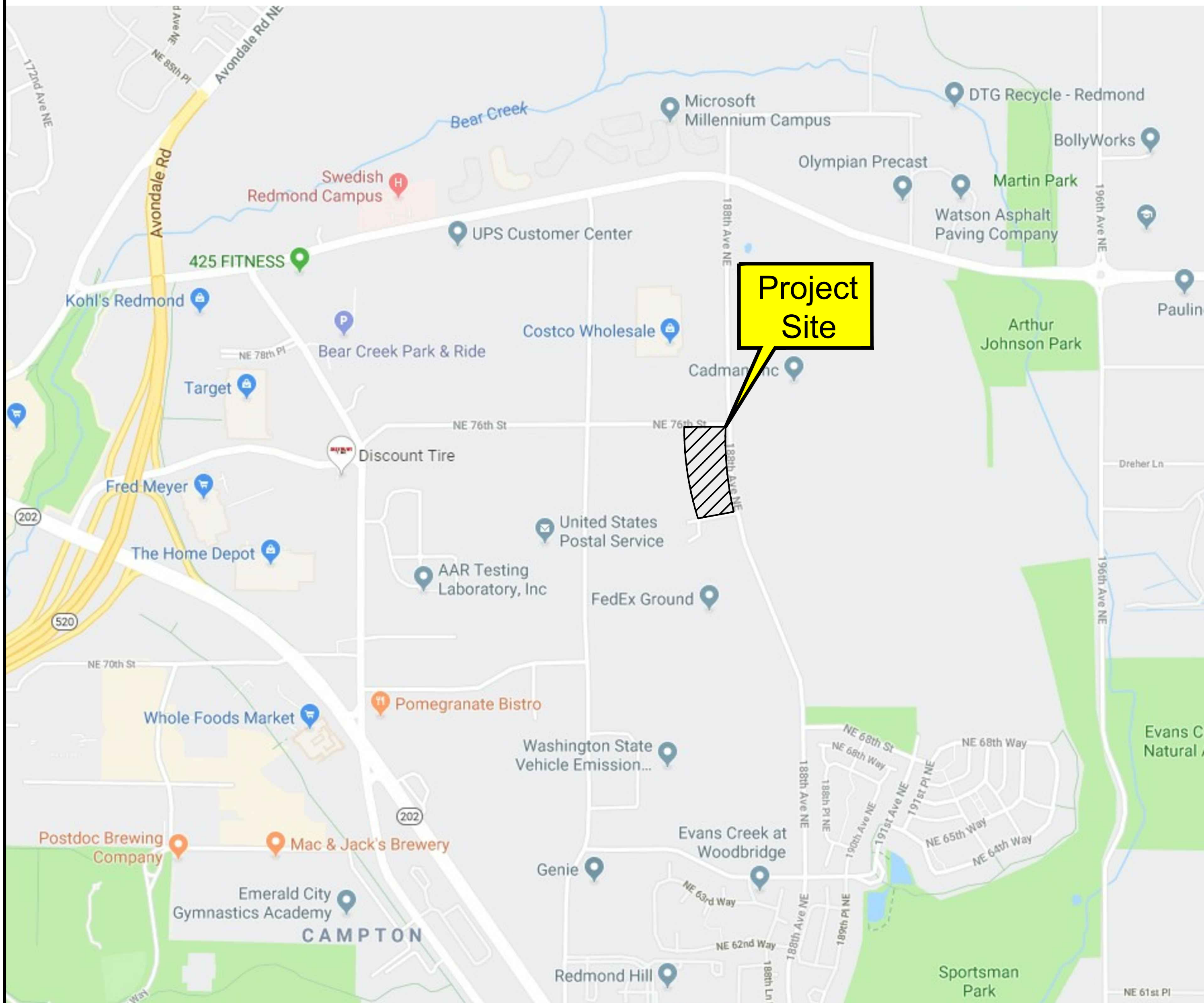
CTC:KMS:dy

Attachments: Six Figures
Appendix A – Keystone Block Retaining Wall Calculations

VICINITY MAP

Not to Scale

N



Redmond, WA

Project Number
9696C19

Figure 1

MV Transportation Facilities Expansion Vicinity Map

Nelson Geotechnical Associates, Inc.
NGA
Geotechnical Engineers & Geologists

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No.	Date	Revision	By	CK
1	8/2/19	Original	DPN	ABR

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Site Plan

LEGEND

Property line

TP-1 Number and approximate location of test pit

0 100 200

Scale: 1 inch = 100 feet

Reference: Site Plan based on an undated plan titled "Topo Plot - Lot 3, Union Hill Corporate Center," prepared by DOWL.

Project Number 9696C19	MV Transportation Facilities Expansion Site Plan	Nelson Geotechnical Associates, Inc. Geotechnical Engineers & Geologists	No.	Date	Revision	By	CK
Figure 2		Woodinville Office 17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax: 481-2510 www.nelsongeotech.com East Wenatchee Office 5526 Industry Lane, #2 East Wenatchee, WA 98802 (509) 665-7696 / Fax: 665-7692	1	8/2/19	Original	DPN	ABR

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UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS MORE THAN 50 % RETAINED ON NO. 200 SIEVE	GRAVEL MORE THAN 50 % OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50 % OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE - GRAINED SOILS MORE THAN 50 % PASSES NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50 %	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50 % OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1) Field classification is based on visual examination of soil in general accordance with ASTM D 2488-93.

2) Soil classification using laboratory tests is based on ASTM D 2488-93.

3) Descriptions of soil density or consistency are based on interpretation of blowcount data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS:

Dry - Absence of moisture, dusty, dry to the touch

Moist - Damp, but no visible water.

Wet - Visible free water or saturated, usually soil is obtained from below water table

Project Number 9696C19	MV Transportation Facilities Expansion Soil Classification Chart	NELSON GEOTECHNICAL ASSOCIATES, INC. NGA GEOTECHNICAL ENGINEERS & GEOLOGISTS Woodinville Office 17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax: 481-2510 www.nelsongeotech.com East Wenatchee Office 5526 Industry Lane, #2 East Wenatchee, WA 98802 (509) 665-7696 / Fax: 665-7692	
Figure 3			

No.	Date	Revision	By	CK
1	8/2/19	Original	DPN	ABR

\\hill\company\2019 NGA Project Folders\9696C-19 Union Hill East Lot 3\Drafting\SC.dwg

LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION
TEST PIT ONE		
0.0 – 8.0		LIGHT BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL, PLASTIC DEBRIS, ORGANICS, AND TRACE COBBLES (LOOSE TO MEDIUM DENSE, DRY TO MOIST) (**RECENT FILL**)
8.0 – 12.0		DARK BROWN TO DARK GRAY, SILTY FINE SAND WITH GRAVEL, ORGANICS, AND WOOD DEBRIS (LOOSE TO MEDIUM DENSE, MOIST) (**HISTORIC FILL**)
		SAMPLE WAS COLLECTED AT 11.5 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 12.0 FEET ON 8/1/2019
TEST PIT TWO		
0.0 – 7.0		LIGHT BROWN TO BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL, CONCRETE DEBRIS, WOOD DEBRIS, AND PLASTIC SCRAPS (MEDIUM DENSE, DRY TO MOIST) (**RECENT FILL**)
7.0 – 13.0		GRAY TO DARK BROWN, SILT WITH FINE SAND TO SILTY FINE SAND WITH GRAVEL, COBBLES, ORGANICS, TRACE WOOD DEBRIS, AND BOULDERS (LOOSE TO MEDIUM DENSE, MOIST) (**HISTORIC FILL**)
		SAMPLE WAS COLLECTED AT 13.0 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 13.0 FEET ON 8/1/2019
TEST PIT THREE		
0.0 – 6.0		LIGHT TO DARK BROWN, SILTY FINE TO MEDIUM SAND WITH DRAIN ROCK POCKETS, BRICK AND CONCRETE RUBBLE, AND TRACE WOOD DEBRIS (LOOSE TO MEDIUM DENSE, DRY TO MOIST) (**RECENT FILL**)
6.0 – 14.0		GRAY, SILT WITH FINE SAND TO SILTY FINE TO MEDIUM SAND WITH DARK BROWN ORGANIC POCKETS, TRACE WOOD DEBRIS, PLASTIC SCRAPS, AND ASPHALT CHUNKS (LOOSE TO MEDIUM DENSE, MOIST) (**HISTORIC FILL**)
		SAMPLE WAS COLLECTED AT 14.0 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 14.0 FEET ON 8/1/2019
TEST PIT FOUR		
0.0 – 7.5		DARK BROWN TO BLACK, SILTY GRAVEL WITH FINE TO COARSE SAND, ASPHALT GRINDINGS, WOOD DEBRIS, ORGANICS, BRICK AND CONCRETE FRAGMENTS, TRACE METAL SCRAPS AND PLASTIC (MEDIUM DENSE TO DENSE, DRY TO MOIST) (**RECENT FILL**)
7.5 – 11.5		GRAY TO GRAY-BLUE, SILT WITH FINE SAND TO SILTY FINE SAND WITH GRAVEL, WOOD DEBRIS, CONCRETE RUBBLE, BRICK DEBRIS, PLASTIC, AND METAL SCRAPS (LOOSE TO MEDIUM DENSE, MOIST TO WET) (**HISTORIC FILL**)
		SAMPLES WERE NOT COLLECTED GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 11.5 FEET ON 8/1/2019

LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION
TEST PIT FIVE		
0.0 – 15.0		DARK BROWN TO BLACK, SILTY FINE TO COARSE SAND WITH GRAVEL, WOOD DEBRIS, CONCRETE RUBBLE, BRICK FRAGMENTS, PLASTIC AND METAL SCRAPS (LOOSE, MOIST TO WET) (**ABANDONED POND FILL**)
		SAMPLES WERE NOT COLLECTED GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 10.0 FEET TEST PIT CAVING WAS ENCOUNTERED FROM 3.0 TO 15.0 FEET TEST PIT WAS COMPLETED AT 15.0 FEET ON 8/1/2019
TEST PIT SIX		
0.0 – 5.5		DARK BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL, ORGANICS, WOOD DEBRIS, AND DRAIN ROCK (LOOSE, MOIST) (**RECENT FILL**)
5.5 – 11.5		GRAY, SILT WITH FINE SAND TO SILTY FINE TO MEDIUM SAND WITH GRAVEL, TRACE ORGANICS, AND WOO DEBRIS (MEDIUM DENSE, MOIST) (**HISTORIC FILL**)
		SAMPLE WAS COLLECTED AT 10.5 FEET GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 5.5 FEET TEST PIT CAVING WAS ENCOUNTERED FROM 3.0 TO 6.0 FEET TEST PIT WAS COMPLETED AT 11.5 FEET ON 8/1/2019
TEST PIT SEVEN		
0.0 – 6.0		GRAY TO DARK BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL AND ORGANICS (LOOSE, MOIST TO WET) (**RECENT FILL**)
6.0 – 11.5		DARK BROWN TO BLACK, SILTY GRAVEL WITH FINE TO COARSE SAND, ASPHALT GRINDINGS, BRICK, WOOD, AND CONCRETE DEBRIS (MEDIUM DENSE TO DENSE, MOIST) (**HISTORIC FILL**)
		SAMPLE WAS NOT COLLECTED GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 11.5 FEET ON 8/1/2019

SPECIFICATIONS FOR REINFORCED WALL

General

1. The contractor shall have an approved set of plans and specifications on site at all times during the construction of the wall. The wall layout is the responsibility of the contractor.
2. Nelson Geotechnical Associates (NGA) should observe and monitor the construction of the wall.
3. Mirafi geogrid 5XT or equivalent shall be used for this project. All geogrid and facing materials shall be approved by NGA prior to installation.
4. The contractor may use longer geogrid lengths than the design sections for ease of construction. The geogrid lengths may not be shorter unless approved by NGA.

Subgrade Preparation

1. The ground should be prepared by removing surficial organics and loose soil to expose competent native soils as approved by the NGA.
2. A generally level bench with a minimum width equal to the design length of the geogrid is required for placement of the reinforced fill.
3. The excavation shall be cleaned of all excess material and protected, as necessary, from construction traffic to maintain the integrity of the subgrade.
4. The base of the excavation should be deep enough to satisfy a minimum embedment of 1.0 feet.

Geogrid Placement

1. The reinforcement shall be rolled out, cut to length, and laid at the proper elevation, location, and orientation. Orientation of the reinforcement is of extreme importance since geogrids vary in strength with roll direction. The contractor shall be responsible for the correct orientation.
2. Geogrid shall be placed at the location and elevations shown on the plans. The geogrid length is measured from the back of the block.
3. Prior to placing the fill, the geogrid shall be pulled to remove the slack and stretched by hand until taut and free of wrinkles.

Fill Placement

1. Structural fill, consisting of granular import soils, would then be placed upon the subgrade and geogrid. If larger rock is used in the fill, additional layers of geogrid may need to be used in the reinforcement. The contractor shall prevent damage to the geogrid by placing the first lift of structural fill with at least a 1-foot thickness. NGA shall approve the material placed in the reinforced zone, before placement.
2. Structural fill should have parameters equal to or better than those stated for the reinforced wall fill below with less then 15 percent passing the number 200 sieve. NGA may allow a higher silt content based on review of the wall design and proposed fill parameters.
3. Soil density tests should be performed as designated by the geotechnical engineer.
4. Fill soils in the wall area shall be compacted to at least 95 percent of the Maximum Dry Density (MDD) as determined by ASTM D-1557.
5. The soil shall be placed in relatively uniform horizontal lifts not exceeding 10 or 12 inches in thickness. The lift thickness shall not exceed the manufacturer's recommended depth for the compactive device used on the project.

Drainage

1. A specific drainage system is shown on the plans. Alternative drains can be used based on conditions found in the field and the material used within the reinforced zone. Changes to the drainage system should be approved by NGA prior to placement.
2. A drainage blanket 12 inches in width should be installed directly behind the keystone block facing and shall consist of 3/4-inch clean crushed rock. All of the drainage materials shall have a fines content no greater than 5 percent passing the number 200 sieve. A 6-inch rigid perforated pipe embedded in a minimum of one foot of pea gravel or washed rock and wrapped with filter fabric should be installed at the bottom of the drainage blanket
3. Surface water shall not be allowed to pond in or near the reinforced fill zone during or after construction.
4. Suitable clean-outs should be installed every 50 feet for future maintenance.

Design Parameters

Reinforced Wall Fill: 34 degrees, 0 PSF, 125 PCF
Retained Backfill: 32 degrees, 0 PSF, 120 PCF
Foundation Soil: 32 degrees, 0 PSF, 120 PCF

External Stability of Wall

Minimum Factor of Safety against Base Sliding: 1.5
Minimum Factor of Safety against Overturning: 2.0
Minimum Factor of Safety against Bearing Capacity: 2.0

Internal Stability of Wall

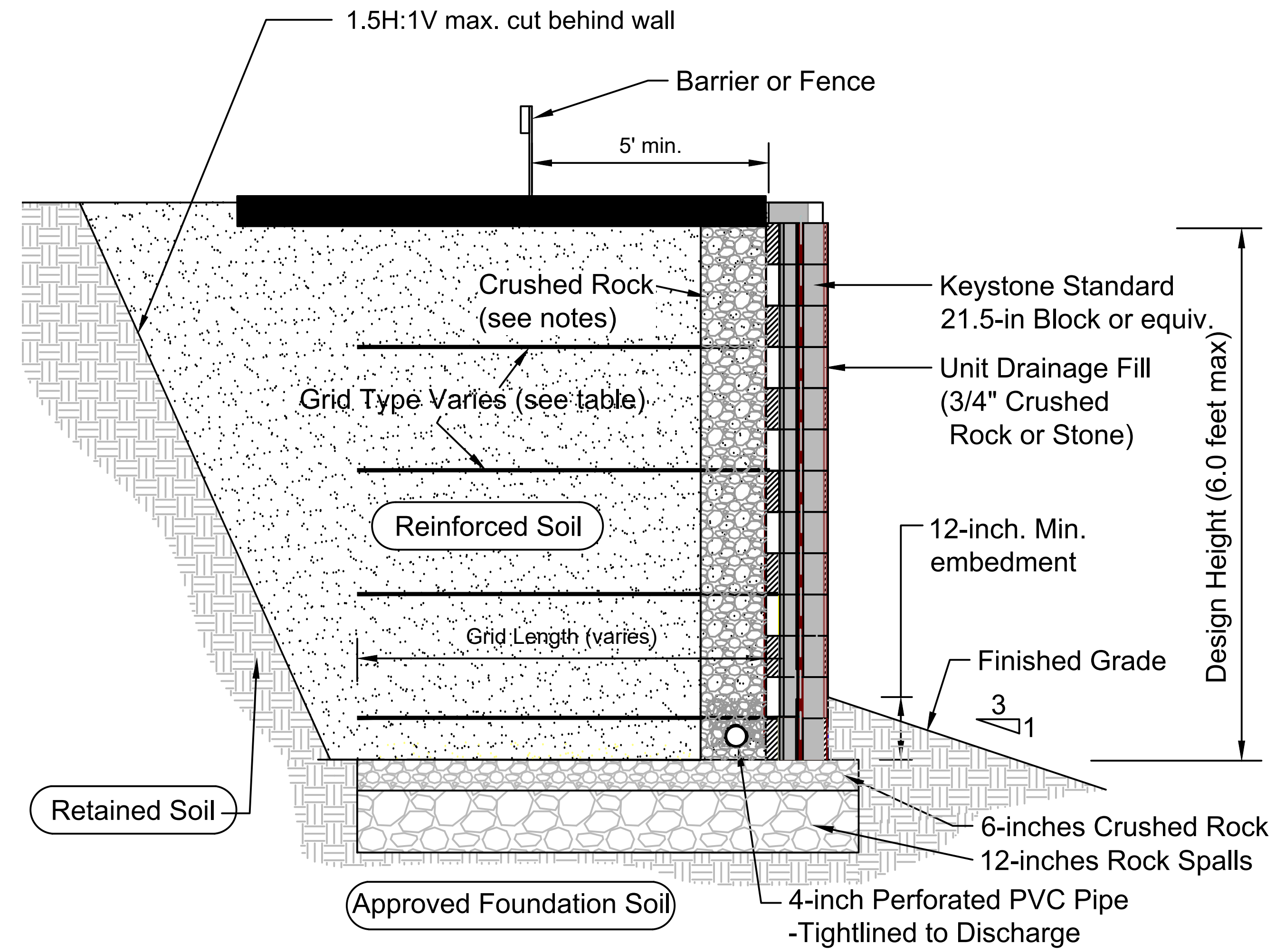
Minimum Factor of Safety on Geogrid Strength: 1.5
Minimum Factor of Safety on Geogrid Pullout: 1.5
Soil-Geogrid Interaction Coefficient: 1.0
Percent Coverage of Geogrid: 100 Percent

External Loading

250 PSF traffic loading located 5 ft from back of wall

Inspection

Wall construction shall be periodically inspected under the direction of NGA.



Wall Height (feet)	Number of Geogrid Layers	Geogrid Length (feet)	Geogrid Height Above Leveling Pad / Geogrid Type (feet)		
4	2	5.0	0.67 / 5XT*	2.67 / 5XT	
6	3	7.0	0.67 / 5XT	2.67 / 5XT	4.67 / 5XT

*Mirafi 5XT Geogrid (or equivalent)

No.	Date	Revision	By	CK
1	8/22/19	Original	DPN	KMS

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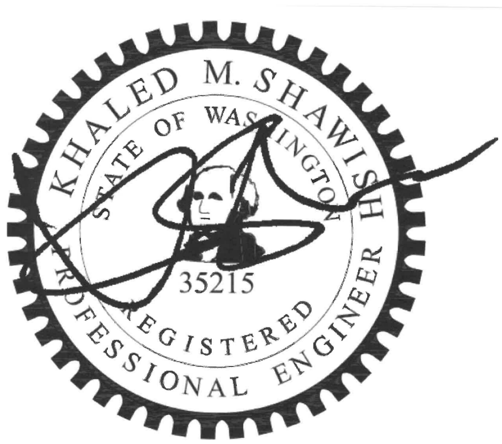
MV Transportation Facilities Expansion
Reinforced-Wall Detail

Project Number 9696C19

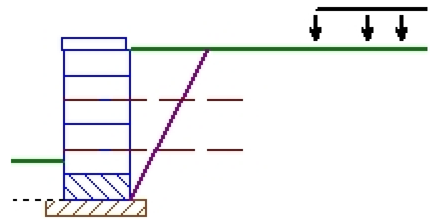
Figure 6

APPENDIX A

Keystone Block Retaining Wall Calculations



Section	4 foot walls
Report Date	August 16, 2019
Designer	Nelson Geotechnical Associates, Inc.
Design Standard	Rankine Theory Analysis
Design	Static and Seismic
Unit of Measure	U.S./Imperial
Selected Facing Unit	Product Line: Keystone Pinned Systems Name: Standard 21
Seismic As 0.25	Default Deflection of 2.00 inch



Soil Parameters

Soil Zone	Phi Angle [degrees]	Cohesion [lb/ft²]	Unit Weight [lb/ft³]	Description
Reinforced	34	n/a	125.00	Sand, Silt, or Clay
Retained	32	0.00	120.00	
Foundation	32	0.00	120.00	
Leveling Pad	40	n/a	n/a	

Section Details

Section Height	4.33	Back Slope	0.00°	LL Surcharge	250	DL Surcharge	0
Design Height	4.00 ft	Crest Offset	0.00 ft	LL Offset	5.00 ft	DL Offset	0.00 ft
Embedment	1.00 ft	Wall Batter	0.00°	Toe Slope	0.00°	Toe Offset	0.00 ft

Minimum Factors of Safety

Reinforced

External		Value	Internal		Value	Facing		Value
FSsl	Base Sliding	1.50	FSsl	Internal Sliding	1.50	FScs	Connection Strength	1.50
FSbc	Bearing Capacity	2.00	FSpo	Pullout	1.50	FSsc	Facing Shear	1.50
FSct	Crest Toppling	1.50	FSto	Tensile Overstress	1.50			
FSot	Overturning	2.00						

Seismic

Reinforced

External		Value	Internal		Value	Facing		Value
FSsl	Base Sliding	1.10	FSsl	Internal Sliding	1.10	FScs	Connection Strength	1.10
FSbc	Bearing Capacity	1.50	FSpo	Pullout	1.10	FSsc	Facing Shear	1.10
FSct	Crest Toppling	1.10	FSto	Tensile Overstress	1.10			
FSot	Overturning	1.50						

Reinforcements

5XT - Miragrid 5XT **Supplier: TenCate Mirafi - Miragrid XT, Fill Type: Clays and Silts**

Tult	4,700.00 lb/ft	RFcr	1.45	RFd	1.15	LTDS	2,684.37 lb/ft
RFid	1.05	Cds	0.70	Ci	0.70		

Connection/Shear Properties

αcs1	687.00 lb/ft	IP-1	1,675.00 lb/ft	αcs2	2,397.45 lb/ft	IP-2	6,000.00 lb/ft
αcs max	2,397.45 lb/ft	au	1,550.00 lb/ft	λu	17.40 lb/ft	Vu(max)	4,709.00 lb/ft

Analysis Results

*** Embedment is included in Bearing Capacity**

External Static	FS			
Bearing Capacity	19.21	Bearing Pressure	560.17	lb/ft²
Overturning	8.34	Max Eccentricity	0.30	ft
Base Sliding	3.29			
Crest Toppling	21.94			
Internal Sliding	6.66			

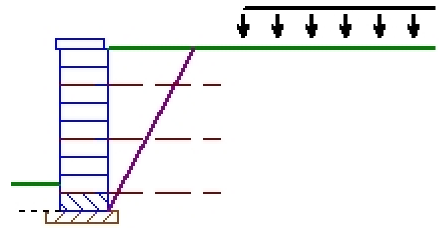
External Seismic	FS			
Bearing Capacity	18.94	Bearing Pressure	568.18	lb/ft²
Overturning	7.56	Max Eccentricity	0.33	ft
Base Sliding	3.17			
Crest Toppling	8.15			
Internal Sliding	12.56			

NOTE: THESE CALCULATIONS, QUANTITIES, AND LAYOUTS ARE FOR PRELIMINARY DESIGN ONLY AND SHOULD NOT BE USED FOR CONSTRUCTION WITHOUT REVIEW BY A QUALIFIED ENGINEER

Internal Static Layer	Elevation	Rein	Length	Load	Tensile Resist.	Tensile FS	Pullout Resist.	Pullout FS	Conn. Resist.	Conn. FS
2	2.67	5XT	5.00	71	2,684	37.98	288	4.08	973	13.77
1	1.33	5XT	5.00	212	2,684	12.66	800	3.77	1,259	5.94

Internal Seismic Layer	Elevation	Rein	Length	Load	Tensile Resist.	Tensile FS	Pullout Resist.	Pullout FS	Conn. Resist.	Conn. FS
2	2.67	5XT	5.00	243	3,892	16.01	288	1.19	973	4.00
1	1.33	5XT	5.00	451	3,892	8.63	800	1.77	1,259	2.79

Section	6 foot walls
Report Date	August 16, 2019
Designer	Nelson Geotechnical Associates, Inc.
Design Standard	Rankine Theory Analysis
Design	Static and Seismic
Unit of Measure	U.S./Imperial
Selected Facing Unit	Product Line: Keystone Pinned Systems Name: Standard 21
Seismic As 0.25	Default Deflection of 2.00 inch



Soil Parameters

Soil Zone	Phi Angle [degrees]	Cohesion [lb/ft²]	Unit Weight [lb/ft³]	Description
Reinforced	34	n/a	125.00	Sand, Silt, or Clay
Retained	32	0.00	120.00	
Foundation	32	0.00	120.00	
Leveling Pad	40	n/a	n/a	

Section Details

Section Height	6.33	Back Slope	0.00°	LL Surcharge	250	DL Surcharge	0
Design Height	6.00 ft	Crest Offset	0.00 ft	LL Offset	5.00 ft	DL Offset	0.00 ft
Embedment	1.00 ft	Wall Batter	0.00°	Toe Slope	0.00°	Toe Offset	0.00 ft

Minimum Factors of Safety

Reinforced

External		Value	Internal		Value	Facing		Value
FSsl	Base Sliding	1.50	FSsl	Internal Sliding	1.50	FScs	Connection Strength	1.50
FSbc	Bearing Capacity	2.00	FSpo	Pullout	1.50	FSsc	Facing Shear	1.50
FSct	Crest Toppling	1.50	FSto	Tensile Overstress	1.50			
FSot	Overturning	2.00						

Seismic

Reinforced

External		Value	Internal		Value	Facing		Value
FSsl	Base Sliding	1.10	FSsl	Internal Sliding	1.10	FScs	Connection Strength	1.10
FSbc	Bearing Capacity	1.50	FSpo	Pullout	1.10	FSsc	Facing Shear	1.10
FSct	Crest Toppling	1.10	FSto	Tensile Overstress	1.10			
FSot	Overturning	1.50						

Reinforcements

5XT - Miragrid 5XT **Supplier: TenCate Mirafi - Miragrid XT, Fill Type: Clays and Silts**

Tult	4,700.00 lb/ft	RFcr	1.45	RFd	1.15	LTDS	2,684.37 lb/ft
RFid	1.05	Cds	0.70	Ci	0.70		

Connection/Shear Properties

αcs1	687.00 lb/ft	IP-1	1,675.00 lb/ft	αcs2	2,397.45 lb/ft	IP-2	6,000.00 lb/ft
αcs max	2,397.45 lb/ft	au	1,550.00 lb/ft	λu	17.40 lb/ft	Vu(max)	4,709.00 lb/ft

Analysis Results

*** Embedment is included in Bearing Capacity**

External Static	FS			
Bearing Capacity	12.66	Bearing Pressure	915.31	lb/ft²
Overturning	5.26	Max Eccentricity	0.57	ft
Base Sliding	2.60			
Crest Toppling	21.94			
Internal Sliding	3.47			

External Seismic	FS			
Bearing Capacity	12.49	Bearing Pressure	928.14	lb/ft²
Overturning	4.97	Max Eccentricity	0.60	ft
Base Sliding	2.54			
Crest Toppling	8.15			
Internal Sliding	4.58			

NOTE: THESE CALCULATIONS, QUANTITIES, AND LAYOUTS ARE FOR PRELIMINARY DESIGN ONLY AND SHOULD NOT BE USED FOR CONSTRUCTION WITHOUT REVIEW BY A QUALIFIED ENGINEER

Internal Static

Layer	Elevation	Rein	Length	Load	Tensile Resist.	Tensile FS	Pullout Resist.	Pullout FS	Conn. Resist.	Conn. FS
3	4.67	5XT	6.00	96	2,684	27.90	278	2.89	973	10.11
2	2.67	5XT	6.00	236	2,684	11.39	1,114	4.73	1,402	5.95
1	0.67	5XT	6.00	375	2,684	7.16	2,452	6.54	1,831	4.88

Internal Seismic

Layer	Elevation	Rein	Length	Load	Tensile Resist.	Tensile FS	Pullout Resist.	Pullout FS	Conn. Resist.	Conn. FS
3	4.67	5XT	6.00	250	3,892	15.59	278	1.12	973	3.90
2	2.67	5XT	6.00	481	3,892	8.09	1,114	2.32	1,402	2.91
1	0.67	5XT	6.00	642	3,892	6.06	2,452	3.82	1,831	2.85